

Total annual costs for treatment rates up to 456 Mgal/d are presented in the following and are plotted in Figure 7-8.

Treatment rate, Mgal/d	Amortized capital cost, \$ million/yr	Operation and maintenance costs, \$ million/yr		Total annual cost, \$ million/yr
		Storage	Treatment	
16.29	1.520	0.144	0.028	1.692
33	1.502	0.138	0.042	1.682
65	1.466	0.126	0.068	1.660
130	1.394	0.102	0.120	1.616
163	1.358	0.090	0.147	1.595
195	1.321	0.077	0.173	1.571
228	1.292	0.066	0.200	1.558
261	1.272	0.055	0.226	1.553
293	1.262	0.045	0.252	1.559
326	1.265	0.036	0.279	1.580
358	1.275	0.028	0.305	1.608
391	1.300	0.021	0.332	1.653
424	1.334	0.015	0.358	1.707
456	1.377	0.010	0.384	1.771

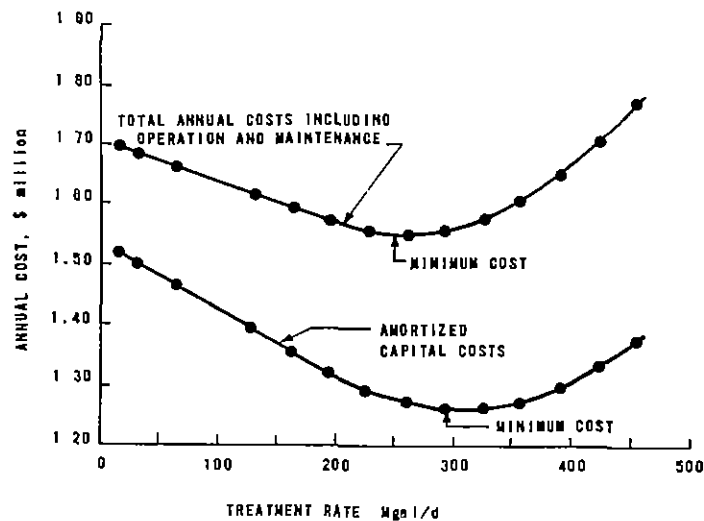


Figure 7-8. Storage/treatment optimization of treatment costing \$30 000/Mgal·d.

- h. Determine the optimum storage/treatment combination. From Figure 7-8, the optimum solution, using capital costs only, is approximately \$12.6 million/yr at a treatment rate of 295 Mgal/d and a storage volume of approximately 4.5 Mgal. With operation and maintenance taken into consideration, the optimum combination is at a treatment rate of 250 Mgal/d with storage at 6.0 Mgal, with a total annual cost of \$1.55 million/yr.

#### Comment

Storage is usually required before treatment of storm and combined sewer overflows to attenuate peak flows and reduce the size of the treatment facility. There is an optimum combination of storage and treatment that produce the least capital cost solution. When operation and maintenance cost is included, the least cost combination shifts to a more storage-intensive solution. Furthermore, as the unit cost of treatment increases, the least cost solution also favors more storage and less treatment. An evaluation of the optimum combinations of storage/treatment at unit treatment costs of \$35 000, \$40 000, and \$45 000/Mgal·d using both amortized capital costs and total annual costs shows that as the unit treatment cost increases, the

optimum treatment rate moves toward the minimum treatment rate of 16.3 Mgal/d. The addition of operation and maintenance costs also shifted the optimum rate toward 16.3 Mgal/d as shown below:

Unit treatment cost, \$/Mgal·d	Optimum treatment rate, Mgal/d	
	Using amortized capital costs	Using total annual cost
35 000	260	195
40 000	220	16.3
45 000	16.3	16.3

#### EXAMPLE PROBLEM 7-5: LAND TREATMENT OF STORMWATER

Determine the land requirements for wetlands, rapid infiltration, and overland flow stormwater land treatment systems. Show the maximum and minimum land requirements based on annual and weekly application rates.

##### Specified Conditions

1. Drainage area = 1000 acres.
2. Runoff coefficient = 0.50
3. Average annual rainfall = 44 in.
4. Use the design criteria shown in Table 118.

##### Assumptions

1. The design weekly rainfall equals the total storm rainfall of 1.6 in. as shown in Figure 7-1.
2. The effects of storage or flow attenuation are not considered in determining the land requirements using the weekly rainfall rate.

##### Solution

1. Determine the annual and the weekly runoff volume from the 1000 acre area.

$$\begin{aligned} \text{a. Annual runoff} &= \frac{(44 \text{ in./yr}) (0.50) (1000 \text{ acres}) (43\,560 \text{ ft}^2/\text{acre})}{12 \text{ in./ft}} \\ &= 79.86 \times 10^6 \text{ ft}^3/\text{yr} \end{aligned}$$

$$\begin{aligned} \text{b. Weekly runoff} &= \frac{(1.6 \text{ in./wk}) (0.50) (1000 \text{ acres}) (43\,560 \text{ ft}^2/\text{acre})}{12 \text{ in./ft}} \\ &= 2.90 \times 10^6 \text{ ft}^3/\text{wk} \end{aligned}$$

2. Determine the maximum and minimum land requirements for wetlands treatment, using design criteria from Table 118.

$$\begin{aligned} \text{a. Maximum land requirement} &= \frac{79.86 \times 10^6 \text{ ft}^3/\text{yr}}{4 \text{ ft/yr}} \\ &= 19.97 \times 10^6 \text{ ft}^2 \\ &\text{or } 458 \text{ acres} \end{aligned}$$

$$\begin{aligned} \text{b. Minimum land requirement} &= \frac{(2.9 \times 10^6 \text{ ft}^3/\text{wk}) (12 \text{ in./ft})}{25 \text{ in./wk}} \\ &= 1.39 \times 10^6 \text{ ft}^2 \\ &\text{or } 32 \text{ acres} \end{aligned}$$

- c. Compute the annual application rate at the minimum land requirement condition.

$$\begin{aligned} \text{Maximum annual application rate} &= \frac{79.86 \times 10^6 \text{ ft}^3/\text{yr}}{(32 \text{ acres}) (43\,560 \text{ ft}^2/\text{acre})} \\ &= 57 \text{ ft/yr} \end{aligned}$$

3. Determine the maximum and minimum land requirements and the maximum annual application rate for a rapid infiltration system.

$$\begin{aligned}
 \text{a. Maximum land requirement} &= \frac{79.86 \times 10^6}{20} \\
 &= 3.99 \times 10^6 \text{ ft}^2 \\
 &\text{or 92 acres} \\
 \text{b. Minimum land requirement} &= \frac{(2.9 \times 10^6) (12)}{120} \\
 &= 2.5 \times 10^5 \text{ ft}^2 \\
 &= 6.7 \text{ acres} \\
 \text{c. Maximum annual application rate} &= \frac{79.86 \times 10^6}{(6.7) (43\ 560)} \\
 &= 273 \text{ ft/yr}
 \end{aligned}$$

4. Determine the maximum and minimum land requirements and the maximum annual application rate for an overland flow system.

$$\begin{aligned}
 \text{a. Maximum land requirement} &= \frac{79.86 \times 10^6}{10} \\
 &= 7.99 \times 10^6 \text{ ft}^2 \\
 &\text{or 183 acres} \\
 \text{b. Minimum land requirement} &= \frac{(2.9 \times 10^6) (12)}{16} \\
 &= 2.18 \times 10^6 \text{ ft}^2 \\
 &\text{or 50 acres} \\
 \text{c. Maximum annual application rate} &= \frac{79.86 \times 10^6}{(50) (43\ 560)} \\
 &= 37 \text{ ft/yr}
 \end{aligned}$$

#### Comment

The ranges of application rates presented in Table 118 were developed for municipal wastewater treatment systems and, therefore, should serve as first-cut guides until more detailed studies using land treatment processes for controlling stormwater are evaluated [78]. These ranges reflect a wide variation in soil types, permeability, slope, climate, and vegetation cover. In this example, the range of annual application rates was narrowed by considering land area requirements based on a design weekly rainfall rate. The land requirements for wetlands range from 3 to 46% of the watershed area. This land, however, would most probably be existing marsh or unusable land areas receiving stormwater discharges directly, or at best an existing marsh operated under a controlled mode of application. Land requirements for rapid infiltration range from 1 to 9%, and for overland flow from 5 to 18% of the watershed area. These land treatment alternatives would require usable or developable land and thus may be limited by land availability and costs.

As with biological treatment systems, overland flow systems were developed for continuous wastewater application to maintain a viable biological mass supported by the grass structure. Because of the intermittent nature of rainfall/runoff, this type of system is reduced to a grass filter for stormwater flows because of the length of time required to develop, stabilize, and sustain a biological mass. Supplemental water may also be required to maintain grass growth during long dry periods. Difficulties may arise with other land treatment methods due to the variability and characteristics of stormwater runoff. Pretreatment may be required for rapid infiltration systems to prevent clogging of the soil by high suspended solids loads.

## SECTION 8

### SYSTEM APPLICATIONS

As has been indicated in previous sections, there is no one single method that is a panacea to all combined sewer overflow or storm drain discharge problems. The size and complexity of urban runoff management programs are such that there is a need for an integrated approach to their solution. The type of problems associated with any given community is dependent upon a number of variables; as a result, the solution for a community must be developed to fit the needs of that particular urban area. The solution is most often a combination of various best management practices and unit process applications.

Important considerations with respect to development and implementation of an urban runoff management program are the regulatory constraints and public attitudes on pollution and environmental objectives that must be met. Often the constraints and attitudes are subject to change with time. This can result in alteration of the ground rules for engineering assumptions so that programs lacking flexibility may be, or in some cases, have been grossly outdated before implementation can be effected. Thus, the political, economic, and environmental constraints affecting an urban runoff management program must be monitored continuously so that the programs can be updated or modified as necessary.

#### CASE STUDY DESCRIPTIONS

The presentation of each stormwater management system application is organized into six parts: (1) problem identification, (2) counter-measure philosophy, (3) design description, (4) cost data, (5) performance and maintenance, and (6) ongoing projects. A variety of system applications are described ranging from major urban metropolitan areas to small suburban communities.

##### Boston, Massachusetts

Combined sewer overflows have contributed to the deterioration of industrial, commercial, and recreational resources of Boston Harbor and the rivers tributary to it [1]. Primary treatment is provided to the intercepted flows at two wastewater treatment plants. However, numerous locations still exist in the Boston Harbor area where, during rainstorms, combined sewage overflows into the receiving waters untreated. These result in bacterial pollution, floating solids, slicks, and sludge deposits.

A wet-weather flow master plan, based largely on preliminary Chicago deep tunnel studies (discussed later in this section), was presented to the City of Boston in 1967 [2]. Four alternatives were studied: (1) complete separation,

(2) chlorination detention tanks, (3) surface holding tanks, and (4) deep tunnels. The deep tunnel alternative was presented because it appeared to offer the best and only feasible method for the complete elimination of overflows. However, following continued review and study of the problems, a demonstration surface detention and chlorination facility was placed into operation in May 1971 at Cambridge, Massachusetts (the Cottage Farm Combined Sewer Detention and Chlorination Station) indicating a viable alternative to the deep tunnel plan.

In 1975, the combined sewer overflow problem was reviewed again in conjunction with the needs for the Boston Harbor-Eastern Massachusetts Metropolitan Area [1]. The major alternatives were (1) sewer separation, (2) overflow diversions via Boston's proposed deep tunnel plan, and (3) intermediate approaches of a decentralized nature. The recommended course of action was to upgrade the two existing treatment plants to secondary treatment and to begin facilities planning for projects identified in the decentralized plan for combined sewer overflow regulation. The decentralized plan would continue present remedial practices and allow piecemeal implementation with immediate opportunities for solving high priority problem areas. The present plan calls for consolidation of the combined sewer outfalls into several groups, each of which would be connected by conduits to transport overflows to regulation facilities for treatment and discharge.

Treatment would consist of several detention facilities located throughout the area where the flow would be stored or, depending on the magnitude of the storm event, detained prior to discharging the overflow. The flow would be disinfected by introducing chlorine upstream from the tanks. The tanks would be designed to provide 15 minutes detention for the peak design flow. The tanks would include floating scum baffles and screens installed between the scum baffle and the overflow weir to polish the overflow before discharge. The stored flow would be returned to the interceptor to receive secondary treatment at one of the two treatment plants.

According to the Report:

...the largest benefits in pollution reduction in decentralized systems will probably come from first flush capture and diversion to the dry weather flow treatment plant and through sedimentation, skimming and disinfection as a result of detaining overflows, while other treatment processes will be employed where such prove to be necessary for further polishing. [1]

The total cost for the various alternatives ranges from \$254 to \$279 million (ENR 2000) excluding projects currently underway (separation in portions of Cambridge and Somerville and construction of the MDC Charles River Chlorination-Detention-Pumping Station Project).

### Chicago, Illinois

In 1967, the Metropolitan Sanitary District of Greater Chicago initiated its wastewater facilities planning study with a 10 year cleanup and flood control program. A major study to develop a comprehensive program for the 972 km<sup>2</sup>

(375 mi<sup>2</sup>) combined sewer area was completed in 1972. The program, presently being implemented, is the Tunnel and Reservoir Plan (TARP). The objectives of the program are:

...to minimize the area's pollutant discharges and the flooding caused by overflows of mixed sewage and wastewater....elimination of the need to release polluted river and canal flood waters into Lake Michigan. [3]

This final TARP is a combination of several alternative plans designed to collect urban runoff during all wet-weather conditions except those storms of a magnitude equal to the three most severe storms recorded to date by the National Weather Service.

Four tunnel systems comprise the TARP. Each tunnel system consists of three components: reservoirs, conveyance tunnels, and sewage treatment plants. A total of three reservoirs, 201 km (125 miles) of conveyance tunnels, and four treatment plants are included in the plan. The combined storage capacity of the plan is approximately 167 750 000 m<sup>3</sup> (44 310 Mgal) of which 11 350 000 m<sup>3</sup> (3 000 Mgal) is tunnel capacity. The total storage capacity is equivalent to 17.3 cm (6.8 in.) of runoff from the combined sewer area, with 1.2 cm (0.46 in.) of runoff capacity in the tunnels alone. The tunnels, located 46 to 88 m (150 to 290 ft) below ground level, range in size from 5 to 10.7 m (17 to 35 ft) in diameter. The total planned treatment capacity will be approximately 96.4 m<sup>3</sup>/s (2200 Mgal/d) of which 91.2 m<sup>3</sup>/s (2150 Mgal/d) is existing. The stormwater treatment rate would be approximately 31.8 m<sup>3</sup>/s (725 Mgal/d) or about 0.5 times average dry-weather flow. More than 640 existing overflow points will be eliminated by the TARP systems. The sub-systems common to all TARP tunnel systems include drop shafts, collecting structures, and pumping stations. Pumping stations will be constructed underground at the end of all conveyance tunnel routes and adjacent to all storage reservoirs. These stations will be sized to allow a full tunnel to be emptied within 2 to 3 days.

In addition, instream aeration at more than ten locations along the Chicago River and Calumet Sag Channel are planned to allow the Illinois standards for dissolved oxygen concentrations to be met.

The Phase I system (tunnels and pumping stations without reservoirs) is under construction currently. The TARP costs are estimated at \$2 553 200 000 (ENR 2000). The breakdown is as follows:

Conveyance tunnels	\$ 869 800 000
Instream aeration	14 000 000
Treatment plant upgrading	986 900 000
Reservoirs and flood control	682 500 000
	<hr/>
	\$2 553 200 000

Additional costs such as sewers, solids disposal, O'Hare Treatment Plant, and non-TARP flood control will raise the total cost to \$2 979 400 000. To date, approximately \$45 000 000 of tunnel construction has been completed and another \$100 000 000 is under construction.

It is projected that the Phase I tunnel system, with overflows at the existing outfalls until the reservoirs are completed, will reduce the number of overflows to the river system to about ten per year. This will result in a 75% reduction in the volume of combined sewage overflowing to the river and a 90% reduction in the combined sewer overflow BOD<sub>5</sub> mass load to the river.

#### Detroit, Michigan

Detroit is served by a combined sewer system and a primary treatment plant. In May 1966, an agreement between the Detroit Metro Water Department (DMWD) and the Michigan Water Resources Commission required

..the City of Detroit to take immediate steps to decrease the frequency, magnitude and pollutional content of all combined sewer overflows from the City's sewer system to the Detroit and Rouge Rivers. [4]

Detroit considered the following alternatives to meet the agreement: (1) systems management utilizing sewer monitoring and remote control of pumping stations and selected regulator gates to affect in-system storage, (2) complete sewer separation, (3) retention basins to capture storm wastewater, and (4) the above in various combinations. After a review of the alternatives, the systems management approach was selected for implementation in a demonstration project [4].

The system developed includes telemeter-connected rain gages, sewer level sensors, overflow detectors, a central computer, a central data logger, and a central operating console for monitoring and controlling pumping stations and selected regulating gates. This system has enabled DMWD to apply such pollution control techniques as storm flow anticipation, first flush interception, selective retention, and selective overflowing.

The in-system storage potential at locations where remote control facilities were installed was 526 500 m<sup>3</sup> (139.1 Mgal). In addition, there is approximately 581 200 m<sup>3</sup> (150 Mgal) of uncontrolled storage in the system.

Upon receiving advance information on storms from remote rain gages, the operator initiates a sewer pumpdown procedure to increase the available in-system storage capacity. This procedure, along with in-system flow routing, has enabled DMWD to contain and treat many intense spot storms entirely, in addition to many scattered citywide rains.

Since the completion of the demonstration project in 1971, DMWD has continued to expand the monitoring project [4]. The change in the system is indicated in Table 122. The supervisory control system has been expanded with the addition of four new control panels in addition to the original three. Remote control facilities including three wastewater pumping stations, four interceptor regulators, three fabridams, two in-system storage gates, one flow routing gate, and one suburban connection have been added. In addition, four suburban retention basins and 11 suburban pumping stations are now displayed.

The DMWD is utilizing sewer system monitoring data to (1) aid in the operation of the system, (2) predict and verify system response to storm events,

(3) establish priorities for overflow abatement projects, and (4) develop computer control algorithms for the various remote control facilities [4]. Additional in-system and offline storage is being investigated.

TABLE 122. COMPONENTS OF THE MONITORING AND REMOTE CONTROL SYSTEM [4]

Item	1971	1975
Rain gages	14	25
Level sensors	118	214
Status sensors	68	110
Pumping stations/pumps	7/39	10/52
Radar remoting	0	1
Regulators	4	10

Cost data for the additions to the monitoring and remote control system were not reported.

#### Milwaukee, Wisconsin

The older areas of the City of Milwaukee are served almost exclusively by combined sewers, approximately 6240 hectares (15 400 acres). Along the Milwaukee River within the City of Milwaukee are 62 combined sewer outfalls. Most of these outfalls, 52, are concentrated in the last three miles of the river before it discharges into Lake Michigan. A flushing tunnel which carries dilution water from Lake Michigan discharges at the head of the reach where the overflows are concentrated. This tunnel has been used since 1888 to dilute the river water to reduce odors.

A demonstration project completed in 1974 studied the concept of detention tanks for attenuating combined sewer overflows. Two of the objectives were [5]:

1. Characterize the performance of a combined sewer overflow detention tank in reducing the pollutional load to the Milwaukee River caused by rainfall in the test area.
2. Project the impact of combined sewer overflow detention tanks on the quality of water in the Milwaukee River.

A 14 760 m<sup>3</sup> (3.9 Mgal) detention tank (Humboldt Avenue Combined Sewer Overflow Detention Tank) serving a 230 ha (570 acre) area was constructed and tested. During the 12 month test period, the tank reduced the volume, BOD<sub>5</sub>, and suspended solids loads from this combined sewer overflow location by 65 to 70%. Studies evaluating detention tank removal efficiencies of BOD<sub>5</sub> and suspended solids indicated that removal due to volumetric retention is much more significant than removals due to sedimentation [5]. Removals due to

sedimentation generally increased total removal efficiency by approximately 5% over removals due to volumetric retention alone.

For purposes of demonstrating the cost impact of the problem, an approximate cost estimate was developed for construction of 13 detention tanks to receive flows from all combined sewer overflow points on the Milwaukee River in the city. These tanks would serve an area of 2350 ha (5800 acre). All tanks would be similar to the Humboldt Avenue facility as far as design criteria are concerned. The implementation of such a series of tanks would be expected to reduce the discharge of pollutants from combined sewer overflows by approximately 80% on an annual basis. The total cost for the facilities would be approximately \$45 050 000. This includes \$28 300 000 for the tanks, \$8 150 000 for pumping stations, and \$8 600 000 for sewers. These costs do not include land, right-of-way, contingencies, or additional treatment facilities.

At the present time, the city is proceeding with the development of a combined sewer overflow abatement program incorporating both detention facilities and other treatment methods.

#### Mount Clemens, Michigan

Combined sewer overflows from the City of Mount Clemens polluting the Clinton River led to a "stipulation" from the Michigan Water Resources Commission in 1967. With regard to combined sewer overflows, the stipulation called for the construction of facilities by June 1972. A demonstration treatment facility was designed to provide treatment to the overflows by means of a series of aerated lakelets with intermediate microscreening, disinfection, and high-rate pressure filtration prior to discharge into the Clinton River [6]. The testing and evaluation of this facility was completed in 1973. One of the conclusions reached regarding the demonstration project was:

The Mount Clemens treatment concept evaluation indicates that it is a feasible and reliable concept....sampling data has demonstrated that the capability of the treatment concept to acceptably renovate combined sewer overflows for fishing and boating and for lawn sprinkling. All water quality parameters, except the toxic and deleterious substances parameter (not studied), were met. [6]

Annual suspended solids and BOD<sub>5</sub> removal efficiencies of about 95% were reported for the demonstration collection and treatment facility.

As a result of the demonstration project findings, the city has developed a citywide project for the abatement of combined sewer overflows. It was recommended that for a 610 ha (1500 acre) portion of the city a combined sewage interceptor be installed to collect the overflows and convey them to a retention basin, the contents of which would be withdrawn at a slow uniform rate for further treatment. For the remaining 240 ha (600 acre) area sewer separation by constructing new collecting sanitary and/or storm sewers was recommended. Construction of the citywide project began in 1974.

The collection and treatment project involves the interception of overflows (5 year storm) from combined sewers and conveying them to the main pumping

station at the retention basin site. The flow will then pass through sedimentation-resuspension chambers before discharge to an aerated retention basin. Any excess will overflow into a chlorination basin before discharge to the Clinton River. Wastewater will be withdrawn from the retention basin at a constant  $0.18 \text{ m}^3/\text{s}$  (4 Mgal/d) rate and conveyed to the existing demonstration project site for treatment. (Dry-weather flow is now treated elsewhere as part of the MACOMB County-Detroit Metro Water Department Regional System.) Treatment will include clarification and disinfection; future chemical additions for phosphate removal will occur at this location. The water will then be discharged to three lakelets in series. The initial lakelet will be an aerated "flow-through" treatment unit. Effluent from the final lakelet will be filtered through high-rate pressure sand filters before discharge to the Clinton River. The city has designated the treatment-park site for development as a recreational facility. The final lakelet is expected to be acceptable for recreational use and potential use for watering park landscaping.

The total construction cost for the sewer separation and the collection and treatment facilities was estimated at \$15 140 000. The sewer separation portion was \$2 160 000. The total project costs (including engineering, legal, fiscal, administrative, and property and easement acquisition) were estimated to be 125% of the construction cost. The treatment facilities are expected to be on-line early in 1977.

#### Rochester, New York

Within the Rochester Pure Waters District, combined sewer overflows represent a major polluttional load to the Genesee River, the Rochester Embayment of Lake Ontario, and Irondequoit Bay. A study completed in late 1976 developed a master plan outlining the actions necessary to achieve a cost-effective solution to the receiving water quality impairment caused by combined sewer overflows [7, 8, 9].

The study was divided into three parts:

- Monitoring and characterization of combined sewer overflows and the collection of field data necessary to characterize the drainage areas serviced by the sewerage system
- Pilot plant study to evaluate the applicability of alternatives
- Application of mathematical models to evaluate the effect of combined sewer overflows on the receiving waters to evaluate the effectiveness of various abatement alternatives [8]

Three classifications of processes were piloted: (1) solids removal; (2) chemical precipitation to achieve a greater degree of fine solids removal along with phosphorus reduction below the 1 mg/L level; and (3) final polishing and high-rate disinfection to achieve a secondary quality effluent with respect to BOD<sub>5</sub> and bacterial contamination. The processes investigated were flocculation/sedimentation with and without chemical addition, microscreening, grit swirl and primary swirl concentrators connected in

series, dual media filtration, carbon adsorption columns, and high-rate disinfection with chlorine and/or chlorine dioxide.

The alternatives investigated included nonstructural alternatives (source control measures and improved sewer system maintenance practices); minimal structural alternatives (improvement of existing dry- or wet-weather storage and treatment facilities); and structural intensive abatement alternatives (new storage and treatment facilities). Mathematical models were applied to evaluate these alternatives. The runoff block of the Storm Water Management Model (SWMM) was used to evaluate the effects of the nonstructural alternatives. Minimal structural alternatives were evaluated using the SWMM transport block. To determine the average annual effect of various abatement measures, the Simplified Stormwater Model was used [8].

The recommended master plan calls for the implementation of interceptor improvements, regulator modifications, blockage of high impacting overflows, addition of control structures, implementation of source control regulations, implementation of an overall control system, construction of wet-weather treatment facilities at the existing Van Lare Treatment Facility (dry-weather flows) site, and inline tunnel storage and conveyance. The cost-effective optimum structural intensive solution based on the 2 year design storm involves a 12.05 m<sup>3</sup>/s (275 Mgal/d) wet-weather treatment capacity and a storage capacity of 227 100 m<sup>3</sup> (60 Mgal). The recommended wet-weather treatment facilities are chemically assisted flocculation/sedimentation (1 mg/L polymer and 40 mg/L alum) followed by high-rate disinfection. The estimated costs associated with implementation of this master plan are \$7 140 000 ± 25% for the nonstructural and minimal structural alternatives and \$88 570 000 ± 20% for the structural intensive storage and treatment alternative [7]. These costs do not include drainage relief facilities that are part of the costs reported in Section 2.

The effectiveness of the proposed master plan was reported as follows:

...incorporating the nonstructural and minimal structural recommendations is projected to reduce the BOD<sub>5</sub> and TKN (total Kjeldahl nitrogen) annual wet-weather loading to the Genesee River from approximately 363 600 kg/yr (800 000 lbs/yr) and 9 090 kg/yr (20 000 lbs/yr) to 1360 kg/yr (3000 lbs/yr) and 114 kg/yr (250 lbs/yr). This will reduce the average annual potential of dissolved oxygen contraventions of the Genesee River from approximately 10 days/yr to 1 day/yr.

...The annual CSO (combined sewer overflow) loading of suspended solids to the Genesee River as a result of implementing the Master Plan will be reduced from approximately 1 363 600 kg (3 000 000 pounds) to a value of less than 4545 kg (10 000 pounds). [7]

### Rohnert Park, California

The City of Rohnert Park has separate sanitary and storm sewers. However, high wet-weather wastewater flows are encountered in the sanitary sewers during the rainy season (October through April). Approximately 95% of the

average annual rainfall occurs during this period. Peak wet-weather flows exceed average dry-weather flows by as much as eight to ten times [10].

A demonstration project, completed in 1973, was undertaken to determine the effect of a surge facility to provide equalized flows to the dry-weather treatment plant. A unique method for maintaining the flow of solids through the basin was tested. One of the objectives of the study was to compare the primary sedimentation tank efficiencies for variable versus uniform flow conditions [10].

The ability of the equalization basin to produce the design uniform flowrate was documented. The basin operated less efficiently than a conventional clarifier for suspended solids and BOD<sub>5</sub> removal due primarily to the variability in the detention time. The BOD<sub>5</sub> removals were quite erratic.

Following completion of the demonstration project Rohnert Park joined in the Laguna Regional Wastewater Treatment Facility. Rohnert Park (including the Town of Cotati and Sonoma State College) is limited to an average dry-weather flow of 0.10 m<sup>3</sup>/s (2.3 Mgal/d) and a peak dry-weather flow of 0.18 m<sup>3</sup>/s (4.1 Mgal/d) to the regional plant. Peak wet-weather flow at the old, existing plant site is 0.53 m<sup>3</sup>/s (12.0 Mgal/d).

The abandoned Rohnert Park treatment plant has been converted to a surge facility for wet-weather flows. The surge facility has a surge basin (old primary sedimentation basin), a storage basin with two days' detention at maximum daily flow, a control building, and a chlorination facility for emergency wet-weather overflow. Most of the components were retained from the abandoned plant. The storage basin is composed of three unlined earthen basins approximately 1.5 m (5 ft) deep with a combined area of 6.9 ha (17 acres). Total storage capacity is 83 300 m<sup>3</sup> (22 Mgal). Flows in excess of 0.18 m<sup>3</sup>/s (4.1 Mgal/d) (are diverted to the surge facility for storage. When the flow in the interceptor to the regional plant falls below 0.18 m<sup>3</sup>/s (4.1 Mgal/d), flow is released from the surge facility. Construction of the surge facility was completed in 1976.

Construction cost for the surge facility was \$943 000. This was composed of \$390 000 for pumping station rehabilitation, \$273 000 for the diversion structure and chlorination facility, and \$280 000 for storage basin earthwork (including regrading and sludge removal from existing oxidation ponds).

### Saginaw, Michigan

The problem at Saginaw was typical of most such systems, namely periodic overflows from the combined sewer system. The distribution of the total intercepted flow among the 34 regulators was inequitable with some contributing a disproportionately large percentage. When flows reached 2.5 times the dry-weather flow, the treatment plant capacity, a valve on the interceptor was closed manually and the flow from one half of the interceptor system was pumped untreated to the river. The valve was reopened manually after the storm when personnel were available. This contributed unnecessarily to the amount of wastes discharged through overflows [11]. In 1969, it was recommended that existing intercepting and stormwater pumping facilities be

utilized to their optimum in conjunction with five new stormwater holding facilities. The holding facilities were to have a storage capacity of  $85\,100\text{ m}^3$  (22.4 Mgal).

In 1972, following application of the Storm Water Management Model (SWMM) to simulate the operation of the sewer system and proposed storage facilities, the plan was revised [12]. The revised plan called for construction of seven storage facilities with a total capacity of  $68\,800\text{ m}^3$  (18.2 Mgal). In addition, revisions to existing regulators would add  $70\,400\text{ m}^3$  (18.6 Mgal) of in-system storage. The size of the required interceptors was also reduced as a result of the SWMM simulations. The sizing is based on the 1-year storm, 4.8 cm (1.9 in.) of rain.

To date, one of the storage facilities is under construction and one about to go to bid. In each facility, as flow enters the covered structure, floating scum and oil baffles rise with the liquid surface to maximize capture of these materials. Depending on the magnitude of the storm, when the basin is filled, effluent passes through horizontal screens (1.25 cm (0.49 in.) mesh) to capture any floatable and suspended material not captured in the settling bays before overflow to the Saginaw River. Influent to the facility is disinfected with sodium hypochlorite. Stored flow is dewatered into the interceptor following the storm.

The capital costs for the entire system (seven storage facilities, regulator modification, etc.) were estimated at \$44 800 000.

The storage facilities are being designed for multiple use. The two facilities designed to date include a multistory parking garage above the storage and treatment basin.

The actual construction cost of the Hancock Street facilities was \$5 216 000 [13]. Approximately 80% of this cost is attributable to the storage facility. The remainder is for the parking garage.

The overall performance of the facilities are estimated to be approximately 30% for  $\text{BOD}_5$  and 50% for suspended solids removal for the design storm. On an annual basis, approximately 90% of the  $\text{BOD}_5$  and 92% of the suspended solids presently discharged to the river would be removed. The basins will completely contain approximately 1.3 cm (0.5 in.) of runoff from the tributary area without overflowing to the river.

#### San Francisco, California

Overflows occur from San Francisco's combined sewer system when rainfall exceeds 0.05 cm/h (0.02 in./h). When rainfall exceeds this amount much of the city's wastewater, sometime as much as  $53\text{ Mm}^3/\text{yr}$  (14 000 Mgal/yr) flows untreated into bay and ocean waters at many points around the city.

A wastewater master plan for an improved wastewater treatment system was developed by the Department of Public Works and its consultants between 1969 and 1974. Since 1974, parts of the plan have been changed as a result of

further design and planning work. As the city proceeds with its 8-year program, further changes are anticipated.

The master plan contemplates the establishment of two treatment plants: a dry-weather flow facility in the southeastern area of the city (San Francisco Bay side) and a combined dry- and wet-weather flow facility in the southwestern area (Pacific Ocean side). Both plants will ultimately discharge to the ocean via a common ocean outfall system. Phase I of the plan is shown in Figure 60 [14].

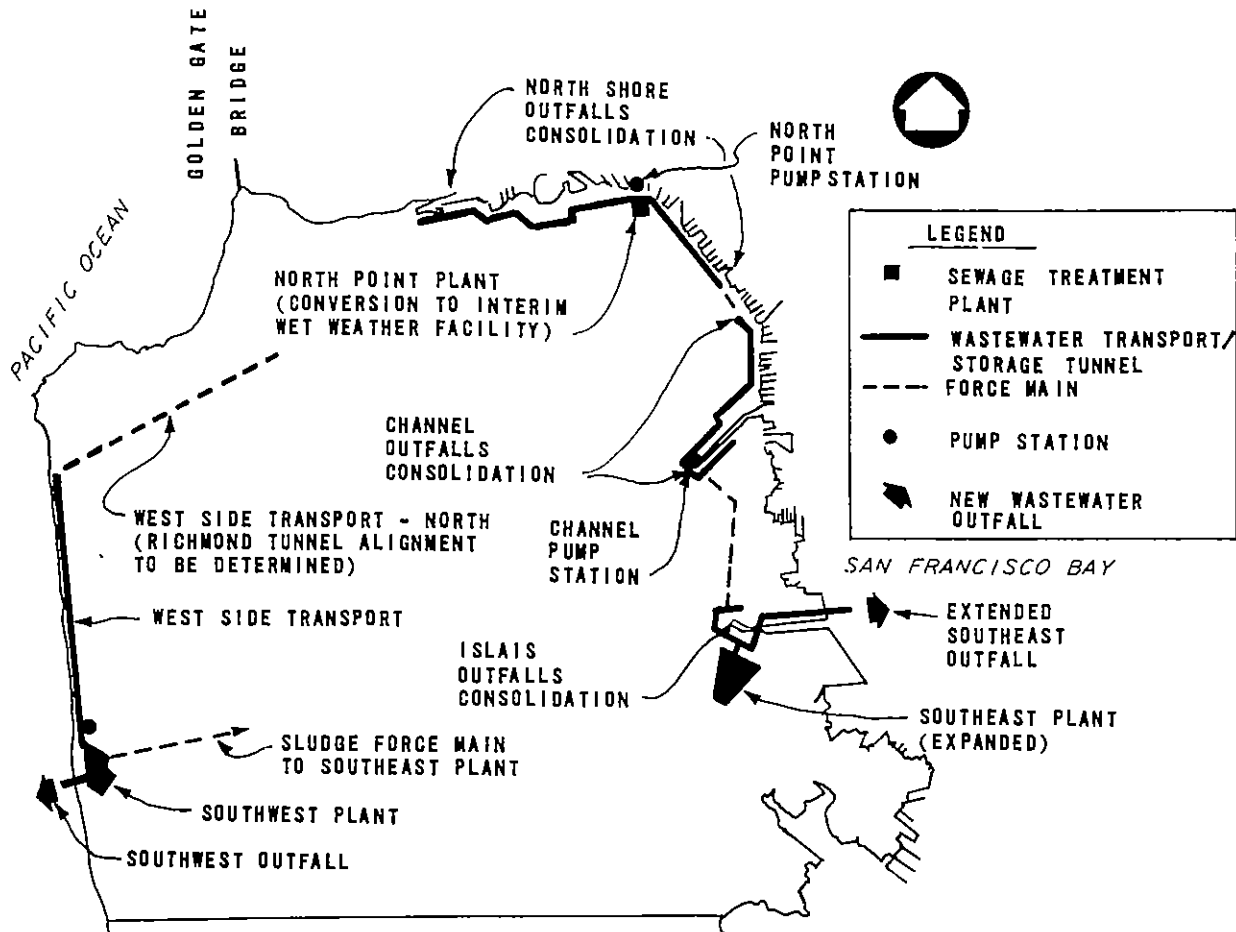


Figure 60. San Francisco wastewater management facilities plan - Phase I.

On the ocean side, the new southwest treatment plant will replace an existing 79 500 m<sup>3</sup>/s (21 Mgal/d) plant. The new plant will treat flows for the western half of the city during wet- and dry-weather. A new outfall, presently under design, will be constructed, which will extend out from the southwest plant approximately 6.4 km (4 mi) offshore. Flows treated at the new plant will be discharged to the ocean through this outfall. A large sewage

transport/storage tunnel and pumping facilities will be constructed along the west side of the city to the new plant.

On the bay side, the existing 71 900 m<sup>3</sup>/s (19 Mgal/d) southeast treatment plant will be expanded to include secondary treatment facilities. The existing capacity will be expanded to 318 000 m<sup>3</sup>/s (84 Mgal/d) to treat all dry-weather flows for the east side of the city. The plant will also handle sludge for the entire city. As an interim measure, the existing 260 000 m<sup>3</sup>/s (65 Mgal/d) North Point treatment plant (dry-weather flows) will be converted to treat wet-weather flows for the northeastern section of the city. No wet-weather treatment facilities are proposed to handle flows from the southeast section of the city during the initial phase of the program.

The large underground interceptor sewers that make up the North Shore, Channel, and Islais outfalls consolidations and the West Side transport will transport dry-weather flows to the treatment plants or pumping stations, and, during storms, store excess wet-weather flows until they can be treated. These facilities, with the exception of the Channel outfalls consolidation, are expected to reduce the number of untreated combined sewer overflows to an average of one per year. The number of overflows in the Channel outfalls area is expected to be reduced to approximately four per year [14].

As part of the long range plan, a crosstown tunnel and expansion of the southwest treatment plant are proposed [15]. Untreated wet-weather flows from the northeast and southeast districts would be transported to the southwest treatment plant in the crosstown tunnel. This tunnel would be designed for both transport and storage. Treatment of wet- and dry-weather flows from the west side and, during periods of storm runoff, excess flows from the east side would be provided at the expanded southwest treatment plant. Wet-weather treatment capacity at the expanded plant will be approximately 35.0 m<sup>3</sup>/s (800 Mgal/d).

The total costs for the first and second stage projects are estimated at \$513 300 000 [15]. The estimated cost for the Phase I portion is \$308 100 000. At the present time, four of ten contracts for the North Shore and Channel outfalls consolidation projects have been awarded. The total bid costs received for these contracts is \$25 700 000 compared to the engineers estimate of \$44 750 000. The estimated cost for this entire consolidation project is \$86 420 000.

A real time automatic control computer program for inline storage and routing control for the North Shore consolidation project is currently under development. The objectives of this program, when ultimately applied citywide, are: (1) minimization of overflows, (2) priority of the location for discharges when overflows must occur, (3) make maximum use of storage facilities, and (4) make optimal use of all facilities [16].

At present design studies for the ocean outfall, expansion and treatment upgrading along with sludge handling at the southeast plant, facilities planning for the new southwest plant, and the West Side transport and pumping station are underway. A feasibility study of the crosstown tunnel is expected to start shortly.

## Seattle, Washington

A comprehensive plan for the collection, treatment, and disposal of wastes from Seattle and other communities within the drainage basin was completed in 1958. Despite improvements brought about by the basinwide construction plan, Seattle itself was still plagued by overflows from the 60-year old combined sewer system. A demonstration project was begun in 1967 to achieve "the ultimate in system storage and control in a combined sewer system through computerized 'total system management'" [17]. This resulted in the development known as the "Computer Augmented Treatment and Disposal System," or CATAD.

The CATAD system is a computer-directed system for maximum utilization of available storage in the trunk and interceptor sewers to reduce or completely eliminate combined sewer overflows. The CATAD system utilizes a computer-based central facility for automatic control of remote regulator and pumping stations. The control center includes a computer, its associated peripheral equipment, an operators console, an interceptor system map display, data loggers, and event printers.

At the same time that the Municipality of Metropolitan Seattle (METRO) was developing the CATAD system, the City of Seattle was proceeding with complete or partial sewer separation projects in several areas of the city. The end result was that the CATAD system serves approximately 5310 ha (13 120 acres) of combined sewers. Of the city's total of 21 060 ha (52 000 acres), the sewer separation area amounted to 7290 ha (18 000 acres).

Remote monitoring and control units were provided to 37 remote pumping and regulator stations. In addition, six remote rain gages are also monitored. The CATAD system can be operated in three different modes: (1) local control, (2) supervisory control, and (3) automatic control. Under local control each station is operated independently by controllers within the station in response to local sensing devices. In the supervisory control mode, stations are operated remotely from the central terminal by the operator via the CATAD computer in response to telemetered data. Stations are operated from the central terminal under program control by the CATAD system computer in the automatic control mode.

Using supervisory control, the volume of overflows was reduced by 35 to 50%. Adding automatic control strategies improved these reductions to over 90% [18]. An optimizing model is being developed that is expected to maintain a performance of at least 80% annual overflow volume reduction. Conclusions reached as a result of the demonstration project include:

Loading analysis reveals that 80 to 90% of the peak loading has been reduced, and the peak loading has been shifted to a higher rainfall rate which occurs less frequently. Total loading in pounds has been decreased an average of 58% for ammonia; up to 76% for COD.

Rainfall intensity has a considerable effect on overflows. Considering the average rainfall rate of a storm, the total system reduced overflow

volumes by 73.6% in supervisory control, 97.2% in automatic control, and 85.8% under combined advanced control modes.

Each station tended to show a "fingerprint" effect for sequential overflow data. This fingerprint was generally unique for each station and usually repeated itself for different storm types. The data indicated that the first flush of materials is often diverted to the interceptor in a combined system rather than overflowing to the receiving water.

Overflow priorities were based primarily upon volume reduction. Station by station priority varied considerably depending on which pollution factor was the basis for establishing priority.

During the course of the study, the Duwamish River receiving water has improved dissolved oxygen content by 1 to 2 milligrams per liter. [18].

The success of the application of total systems management concepts is aided by the improved surveillance afforded by the continuous monitoring capability. But the greatest part of the improved performance is due to the ability (under either supervisory or automatic control) to locate portions of the sewer system which can be utilized for storage, thereby allowing overburdened portions of the system to flow more freely [18].

The modifications to the existing combined sewer system included combined sewer separation work by the City of Seattle affecting about 25% of the combined sewers in the CATAD area; modifications to and construction of regulator and pumping stations by the City of Seattle; modification of regulator stations required for CATAD by METRO; and acquisition and interfacing of the telemetry system, controls, and computer for CATAD by METRO. The total cost for the modifications and acquisitions was \$165 650 000. The cost associated with just the CATAD system (regulator station modifications, telemetry system, and control and computer equipment) was \$8 390 000. These costs on a unit area basis were \$5110/ha and \$260/ha (\$12 625/acre and \$640/acre), respectively.

### The Woodlands, Texas

A new town, The Woodlands, is under development 56 km (35 mi) north of Houston, Texas. The town will contain all services of a modern city, including facilities for social, recreational, education, commercial, institutional, business, and industrial pursuits. When development began in 1972, the 7200 ha (17 780 acres) was just heavy forest. Development will span 20 years and lead to homes for approximately 150 000 people.

The basic drainage system planned for The Woodlands was designed on the basis of what was termed the "natural drainage" concept. This concept consists of the following principles:

- (a) the existing drainage system in its unimproved state is utilized to the fullest extent possible; (b) where drainage channels need to be constructed, wide, shallow swales lined with existing vegetation are used

instead of cutting narrow, deep ditches; (c) drainage pipes and other flood control structures are used only where the natural system is inadequate to handle increased urban runoff, such as in high-density urban activity centers; and (d) flow retarding devices such as retention ponds and recharge berms are used where practical to minimize increases in runoff volume and peak flow rates due to development. [19]

It was originally estimated that utilizing the "natural drainage" concept would keep the drainage system costs down to about 50% of that for conventional systems. As part of the initial planning, the impact of the planned urbanization in The Woodlands community was evaluated using the Storm Water Management Model (SWMM). The results were used to develop a program to minimize impact of further development.

To minimize the amount and rate of increased runoff due to urbanization, existing drainage courses are grass covered to slow and reduce runoff through infiltration. Storage reservoirs are used to promote recharge of groundwater and attenuate runoff. Examples of the use of natural drainage features and storage reservoirs are shown in Figure 61. Erosion control measures in construction areas minimize solids loadings in runoff from these areas. The type and amount of fertilizers, pesticides, and herbicides are controlled to minimize pollution of runoff [20].

The Woodlands terrain in many places is quite flat. In a recent review it was reported that in such spots, natural drainage has been found to cause flooding of homesites [21]. Also, Houston area officials dislike the natural drainage idea--drainage swales and ditches accumulate debris and silt, and bushes grow there. Removing the debris and bushes is a maintenance cost. These officials feel sewers are less of a problem. The goal is still to use natural drainage wherever practical, but to balance ecology with practical economics since no one wants to live on flooded land.

Part of the original intent was to provide multifamily and cluster housing to keep the developed land to a minimum, thus minimizing the increased runoff from urbanization. However, many Houstonians who can afford new housing want single-family housing [21]. This may result in a smaller percentage of The Woodlands land left in open space than was originally planned. This would most likely increase the amount and rate of runoff.

## SUMMARY

From the case studies presented and summarized in Table 123, it is apparent that all use an integrated approach toward solving the stormwater pollution problems. The programs developed by communities with combined sewers generally rely on structural methods to solve the overflow problems. For communities with separate sewers, the stormwater abatement programs incorporate both best management practices and structural solutions. This difference in approaches is probably best explained by comparing the types of communities with combined or separate sewers.

Most of the combined sewers are found in the older, highly urbanized cities. As a result, the more easily implementable and least costly best management



(a)



(b)



(c)

Figure 61. Natural drainage and storage reservoir, The Woodlands, Texas.  
 (a) and (b) Natural drainage swales. (c) Stormwater storage basin used  
 as a recreational reservoir in planned development.

TABLE 123. COMPARISON OF CASE STUDIES IN VARIOUS CITIES

Item	Boston	Chicago	Detroit	Milwaukee	Ht. Clemens	Rochester	Bohnert Park	Saginaw	San Francisco	Seattle	The Woodlands
Year study completed	1975	1972	1974	1974	1973	1976	1973	1972	1974	1973	....
Avg annual rainfall, in	42.0	33.2	30.9	27.6	27.4	31.6	29.3	28.4	20.8	38.9	45.3
Avg summer rainfall (May-Sep), in	16.9	17.1	14.5	14.6	12.9	13.4	1.65	14.2	1.0	7.2	20.8
Design storm	1 yr, 6 h	...	..	....	5 yr	2 yr, 2 h	...	1 yr	..	....	....
Type and total sewerage area, acres	Combined 17 000	Combined 240 000	Combined 688 000	Combined 5800	Combined 2100	Combined 12 000	Separate 12 800	Combined 10 200	Combined 24 000	Combined 13 120	Separate 17 780
Storage volume, Mgal	21 6-39.3	Tunnels 3000 Total 44 320	Controlled 139 1 Total 293	40.4	Retention basin 33.2 Lakelets 18.9	0.79	22	Tanks 18.2 Total 36.8	222.6	17.8	....
Storage volume, in. of runoff	0.05-0.09	0.46 tunnels 6.80 total	0.01	0.26	0.79	0.18	0.06	0.5	0.34	0.05	....
Treatment rate, Mgal/d	....	1100 (storm-water)	....	....	4.0	275 (stormwater)	1.8 maximum	320	800 (storm-water)	....	...
Treatment rate as multiple of BAF (includes BAF)	....	1.5	....	....	NA	3.75	1.8	....	8.0	3.0	....
Available treatment	Primary	Secondary	Secondary for stored flows	Secondary	Secondary + filtration	Chemically assisted primary	Secondary	Primary (overflow) Secondary (stored)	Advanced primary	Secondary	....
Problem	CSOs to Massachusetts Bay and Charles River.	CSOs to Chicago River, flooding in downtown area.	CSOs to Detroit and Rouge rivers and Lake St. Clair.	CSOs to Milwaukee River.	CSOs to Clinton River	CSOs to Genesee River and Lake Ontario	Heavily infiltrated sanitary sewers.	CSOs to Saginaw River	CSOs to San Francisco Bay	CSOs to Lake Washington, Elliott Bay, and Duwamish River.	Avoid change in runoff rate and volume. Minimize development impact.

TABLE 123 (Concluded)

Item	Boston	Chicago	Detroit	Milwaukee	Mt. Clemens	Rochester	Rohmert Park	Saginaw	San Francisco	Seattle	The Woodlands
Solution	Chlorination with a minimum of 15 min detention. Screening and skimming of floatables. De-water to Interceptor.	Deep tunnels for storage and transport. Reservoir storage. Upgrade existing STPs. In-stream aeration in Chicago River.	In-system storage. Selective overflowing. Remote control of pumping stations and gates.	Storage/detention with disinfection at 13 locations. De-water to Interceptor.	Sewer separation for 500 acres. Treatment for 1600 acres. Chemical addition and high-rate disinfection. Sludge storage, sedimentation, disinfection, and filtration.	In-line storage in transport tunnel. Flocculation/separation. Chemical addition. High-rate disinfection.	Pumping station rehabilitation. Litigation. Convert existing oxidation ponds to storage DMF ponds to regional STP.	In-line storage and detention tanks with disinfection and effluent screening. Parking garage above tanks.	In-line and off-line storage. Computer control of regulator system. Treatment of stored flow.	Partial separation. In-line storage. In-line storage and computer control of regulator system. Storm sewer construction. Use porous pavements and percolation.	Utilize natural drainageways. Surface storage. Detention ponds. Minimize storm sewer construction. Use porous pavements and percolation.
Capital cost, \$ x 10 <sup>6</sup> <sup>a</sup>	254-279	Tunnels 870.0 Total 2553	....	45.0	Separation 2.2 Treatment 13.0 Total 15.2	95.7	0.943	44.8	513.	CATAD System 8.4 Total 165.6	....
Capital cost, \$/acre <sup>a</sup>	14 900 to 16 400	Tunnels 4400 Total 12 600	..	7770	Separation 3600 Treatment 8650 Total 12 210	7980	74	4400	21 375	CATAD System 640 Total 12 625	....

<sup>a</sup>. Based on EIR Construction Cost Index = 2000.

acre x 0.405 = ha

in. x 2.54 = cm

Mgal x 3785 = m<sup>3</sup>Mgal/d x 0.0438 = m<sup>3</sup>/s

practices such as onsite retention, erosion control, use of pervious areas for percolation, and use of natural drainage features to attenuate runoff are difficult, if not impossible, to apply. Thus, reliance on structural methods such as storage and treatment is necessary. Separate sewers may be found in the newer portions of some old cities and in suburban communities. In these areas, best management practices are usually more easily implemented. Incorporating best management practices into the stormwater abatement program generally reduces the need for structural solutions.

It is noteworthy that all of the programs incorporate storage in one form or another. This allows a greater stormwater volume to be treated than just relying on the interceptor capacity to convey stormwater to a treatment plant. In most cases, inline storage is included; even where offline storage is used. This allows the stormwater to be treated using the excess capacity at existing treatment plants or allows the use of smaller new treatment plants.

The unit capital costs for the programs range from \$1780/ha to \$8660/ha (\$4400/acre to \$21 375/acre) for communities with combined sewers. There are insufficient data to determine a similar range of costs for communities with separate sewers. Direct comparison of the unit costs for the sewer separation and collection/treatment options for Mount Clemens should not be made since separation is being done in an area that is primarily industrial and open space. The costs for collection and treatment of the combined sewer overflows (in areas where this option was selected) were approximately 30 to 60% of the cost for sewer separation in the same areas.

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